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# DEVELOPMENT OF SEISMIC DESIGN METHODOLOGIES FOR RIGID WALL-FLEXIBLE DIAPHRAGM STRUCTURES

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## ABSTRACT

Evidence indicates that the dynamic behavior of Rigid Wall – Flexible Diaphragm (RWFD) structures is dominated by the diaphragm's response instead of the walls' response, and this is a significant departure from the underlying assumptions of the widely used equivalent lateral force method in current building codes. RWFD buildings are common in North America and other parts of the world, and incorporate rigid in-plane concrete or masonry walls and flexible in-plane wood or steel roof diaphragms. With the use of a numerical computer modeling framework developed specifically for this type of building, this study sets out to investigate the seismic response of a variety of building archetypes with the intent to develop a simpler, more rational approach to the engineering design of RWFD buildings. A representative list of building archetypes is developed accounting for a variety of common parameters found in North America involving the building size, shape, diaphragm material, and diaphragm connections. Archetype designs are developed under ASCE/SEI 7-10 and this study's proposed approach to develop design methodologies uses the FEMA P-695 methodology to evaluate building performance. In addition, two separate seismic force levels were utilized during the assessment representing both moderate and high seismic exposures to evaluate the impact of these parameters.

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Evidence indicates that the dynamic behavior of Rigid Wall – Flexible Diaphragm (RWFD) structures is dominated by the diaphragm's response instead of the walls' response, and this is a significant departure from the underlying assumptions of the widely used equivalent lateral force method in current building codes. RWFD buildings are common in North America and other parts of the world, and incorporate rigid in-plane concrete or masonry walls and flexible in-plane wood or steel roof diaphragms. With the use of a numerical computer modeling framework developed specifically for this type of building, this study sets out to investigate the seismic response of a variety of building archetypes with the intent to develop a simpler, more rational approach to the engineering design of RWFD buildings. A representative list of building archetypes is developed accounting for a variety of common parameters found in North America involving the building size, shape, diaphragm material, and diaphragm connections. Archetype designs are developed under ASCE/SEI 7-10 and this study's proposed approach to develop design methodologies uses the FEMA P-695 methodology to evaluate building performance. In addition, two separate seismic force levels were utilized during the assessment representing both moderate and high seismic exposures in order to evaluate the impact of these parameters.

## Introduction

The current building code provisions most frequently used in the United States represent seismic response based on a classical model that is quite different from the actual seismic behavior of Rigid Wall-Flexible Diaphragm (RWFD) buildings. The past seismic performance of these buildings has been troublesome, and the code requirements for these buildings have evolved mostly as reactions to observed damage with little consideration of how these buildings respond differently to earthquakes than multi-story buildings or one-story buildings with rigid diaphragms. These buildings have diaphragms that dominate the building behavior; yet due to their complex inelastic response, past attempts in accurate modeling have typically been time consuming and elusive. With a numerical modeling framework developed specifically for this building type and that balances numerical efficiency and accuracy, the development of new

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seismic design methodologies for RWFD buildings may be possible to simplify the design process and provide a more rational design approach.

### **Typical RWFD Structure Description**

Structures containing rigid walls with flexible diaphragms are ubiquitous across our urban environment. Often labeled as “big-box” buildings, these structures are the mainstay for retail, storage and distribution facilities for North America’s largest companies. These buildings are favored by developers and owners for providing the most cost effective approach to enclosing large floor spaces while providing durable and secure perimeters.

RWFD buildings incorporate concrete or masonry walls, which are considered rigid in-plane, with flexible horizontal in-plane steel or wood roof diaphragm systems. These rigid walls act as shear walls to provide seismic shear resistance. Concrete wall systems are most often tilt-up concrete, a unique form of site-cast precast concrete [1]. These highly efficient and versatile enclosures are common across North America, including in high seismic zones. Plant-cast precast concrete walls and concrete block masonry are also very popular perimeter shear wall systems enclosing these structures. These rigid wall systems inherently carry large perimeter seismic weights relative to the roof diaphragm weight.

Roof diaphragms in these buildings consist either of a steel deck diaphragm or a wood structural panel diaphragm depending upon the regional preferences. Steel deck diaphragms are most popular in Canada, Mexico, as well as the Eastern, Central and Southern United States; and the corrugated steel decking is fastened to supporting steel joists (open-web trusses) with welds or screws, and sometimes with an assortment of proprietary fasteners. The in-plane shear strength and stiffness of these diaphragms are a function of the steel deck gage, joist spacing, and fastener type and spacing [2]. Unlike composite steel decking topped with concrete, a popular floor and roof system in multistory buildings, untopped steel deck diaphragms are relatively flexible in-plane.

Diaphragms consisting of wood structural panels are very common in the Western and Southwestern United States, especially in high seismic regions. Plywood, or more often oriented strand board (OSB), is fastened with nails to wood framing to provide a structural diaphragm as well as a roofing substrate. More commonly encountered today, these wood structural panels are fastened to wood nailer plates that are factory installed on top of steel joist and joist-girder roof support structure. The speed and cost-efficiency of combining the wood-based diaphragm with a steel support structure make this “hybrid” system very popular in RWFD buildings in California, Oregon, Washington and Nevada. The in-plane shear strength and stiffness of these diaphragms are a function of the wood structural panel thickness and grade, as well as nail size and spacing [3]. Similar to steel deck systems, wood structural panel diaphragms are also relatively flexible and lightweight compared with the surrounding rigid walls.

### **Past Seismic Performance**

Historically, the seismic performance of RWFD buildings has been poor. These buildings typically suffer from the poor performance of the out-of-plane anchorage that attaches the heavy

walls to the lightweight roof diaphragms. Observed damages and instrument records from the 1971 San Fernando, 1984 Morgan Hill, 1989 Loma Prieta and 1994 Northridge earthquakes have continually led to improved building code provisions for wall anchorage [4]. Based on observations following the Northridge earthquake, the current wall anchorage provisions referenced by the 2012 International Building Code [5] are contained in ASCE/SEI 7-10 Section 12.11.2 [6] and prescribe maximum expected design forces without allowing reliance on connection ductility [7]. These design force levels and detailing requirements for out-of-plane wall anchorage have remained largely unchanged since they were introduced into the 1997 Uniform Building Code. Since that time, the current practice and force levels of anchoring heavy walls to the flexible diaphragms have not been tested by a strong earthquake event.

Earthquake damages to the in-plane rigid shear walls or the main flexible roof diaphragm have been rare, except for collateral damage from the out-of-plane wall anchorage issues. The perimeter shear walls often consist of largely solid wall portions with relatively few penetrations, resulting in excessive lateral strength. This inherent overstrength of the in-plane shear walls combined with the new wall anchorage and diaphragm collector design forces factored up to maximum expected levels is now expected to transfer the inelastic building behavior into the diaphragm, making this the more critical element in the seismic force-resisting system (SFRS). Unfortunately, because diaphragm performance is primarily attributed to the performance of the fasteners installed from above, damage is often hidden without invasive access through the roofing assembly following a significant earthquake. It is important to consider that the out-of-plane detachment of the heavy walls from the diaphragm in the past may have been protecting the diaphragm from experiencing the shear forces which could have led to global failure.

### **Current Building Code Seismic Provisions**

While the out-of-plane wall anchorage provisions have dramatically evolved after each damaging earthquake, code complying design methods have remained fairly consistent. For RWFD buildings, the past and current practices are to engineer the SFRS using the equivalent lateral force (ELF) procedure of ASCE/SEI 7-10 Section 12.8. This procedure assumes the predominant structural response is closely associated with the vertical elements of the SFRS. Under the ELF procedure, seismic forces are a function of evaluating lumped masses of various story levels supported on flexible elements, which represent the lateral stiffness of shear walls or frames traditionally defining the SFRS. The structural system's period, which is based on the structure's height, is the key to determine the code-based seismic forces.

Unfortunately, this simplistic model assumed for the ELF procedure fails to capture the actual behavior of RWFD buildings. The ELF procedure assumes seismic response consists primarily of deforming vertical elements and that the horizontal diaphragm is rigid, i.e. deformation of the diaphragm is not considered. However, for most RWFD structures the primary seismic response is deformation of the horizontal flexible diaphragm instead of the rigid vertical walls. A more accurate structural model would capture the flexible diaphragm dominating the response.

The ELF procedure also inappropriately assumes that the primary inelastic response is in the vertical wall or frame system, instead of the roof diaphragm. The seismic response

modification factor,  $R$ , in ASCE/SEI 7-10 Table 12.2-1 is used to estimate the strength demands on systems that are designed using linear methods while responding in the nonlinear range. Currently, the selection of  $R$  for design is solely based on the performance characteristics of the SFRS's vertical elements. The inelastic response is likely to be in the horizontal diaphragm, therefore the current ELF procedure fails to characterize this diaphragm property adequately. Currently, wood and steel flexible diaphragms and rigid concrete diaphragms are all designed as if they have the same inelastic response.

Because RWFD buildings typically have excessive strength in the shear walls as compared with the diaphragm, it can be unrealistic to expect (or require) the failure mode to be in the walls instead of the diaphragm; despite the fact that the response modification factor  $R$  is selected based on that assumption. Past failures have typically included out-of-plane wall detachments. However, as that failure mode becomes more under control, it is expected that diaphragm damage will be the more dominant form of inelastic behavior, which cannot be captured by the current ELF procedure.

### **Development of New Seismic Design Methodologies**

With seismic response dominated by the horizontal diaphragm in RWFD structures, it is more realistic that design forces are developed around the diaphragm's behavior. Furthermore, because the inelastic behavior in a RWFD building is typically located in the diaphragm, it is more realistic that the seismic system capacity is developed around the diaphragm's overstrength and ductility instead of the vertical elements of the SFRS. A new approach based on the diaphragm's response would be more realistic and has the potential to better evaluate a structure's margin against collapse.

### **Lateral Loads Applied to the Structural System**

A more accurate method of establishing lateral design loads for buildings dominated by diaphragm response is to consider the diaphragm's period relative to the design spectral acceleration. As an example, ASCE 41-06 [8] provides the *Linear Static Procedure* to rehabilitate existing one-story RWFD buildings by estimating the diaphragm-dominated building period and then establishing a pseudo-lateral force on the system. A number of other sources have proposed other methods of estimating flexible diaphragm periods and their corresponding pseudo-lateral force [4, 9, 10]. With a more accurate period of the dominating response, the force-based procedures of ASCE/SEI 7 can be used more appropriately. Furthermore, the use of an ELF approach with a response modification factor  $R$  related to the diaphragm construction and detailing instead of the vertical elements of the SFRS is expected to produce more rational results.

### **Designing the Whole Structural System Systematically**

Both the inelastically acting horizontal diaphragm and the SFRS's vertical elements need a unified design methodology. The fact that RWFD buildings have a flexible upper portion supported in series with a rigid lower portion make them ideally suited to be designed with a two-staged ELF procedure similar to what already exists within ASCE/SEI 7-10 Section

12.2.3.2. This approach often used for podium type structures allows the two portions to be treated independently and together as appropriate. As the seismic forces are handed off from one portion to another, they are adjusted either up or down to reflect the next portion's expected seismic performance influenced by its period and stipulated R-factor. For simplicity, it is reasonable to develop a methodology for RWFD structures that can fit within the existing framework already familiar to practitioners, yet providing a more rational approach than currently exists.

### **Methodology Validation**

Historically, validation of seismic design methodologies was simply based on field reconnaissance following major earthquakes. While learning from earthquakes is invaluable when validating current design practices, a new proposed design approach needs a more systematic form of validation. The methodology contained in FEMA P-695 [12] is intended to provide a means to evaluate a SFRS proposed for adoption into building codes, but can also be used to evaluate proposed design methodologies.

### **FEMA P-695 Evaluation**

The FEMA P-695 methodology is used to reliably quantify building system performance and provide guidance in the selection of appropriate design criteria when ASCE/SEI 7 linear design methods are applied. The primary objectives of FEMA P-695 are to obtain an acceptably low probability of collapse of the SFRS under maximum considered earthquake (MCE) ground motions, and to provide a uniform protection against collapse across various structural systems. An appropriate P-695 evaluation must have a representative nonlinear model that includes both detailed design information of the system as well as comprehensive test data on the post-yield performance of system components and subassemblies.

A proposed structural system, or as in this case a proposed design methodology of an existing system, is evaluated through the use of collapse fragility curves with collapse margin ratios evaluated in conjunction with the uncertainties judged to be within the evaluation process. Incremental Dynamic Analyses (IDA) [13] on a representative ensemble of nonlinear numerical building models spanning the design space are conducted to build the fragility curves using a pre-determined ensemble of earthquake ground motions. The number of archetypes selected is based on building an appropriate representation of the typical RWFD structure including the range of variation reasonably expected and likely to affect performance.

### **RWFD Archetypes**

Using professional engineers and researchers with expertise in RWFD structures across North America, a list of significant parameters was established. Two of the most significant parameters have already been discussed: steel deck diaphragms and structural wood panel diaphragms. Within the steel deck parameter, several varieties of fasteners are commonly used, each with different nonlinear properties. Depending upon the geographic region, welds, screws, pins, button punches, and various proprietary seam attachments are utilized to varying degrees. Other important parameters are diaphragm size and aspect ratio. Diaphragms with a horizontal clear

span of up to 400-ft and aspect ratios up to 2:1 were considered. In practice, buildings larger than this located in areas subjected to higher seismicity generally have an interior shear element introduced creating several smaller adjacent diaphragms to keep the diaphragm shears manageable.

Combining the different parameters into various performance groups created the basis for a number of archetypes to be evaluated for their risk of collapse under current code and potential future design methodologies. Because RWFD building stock is common across North America, ground motions associated with both Seismic Design Categories (SDC)  $D_{max}$  and  $C_{max}$  were evaluated. It was judged by the authors that ground motions for SDC A and B regions of North America were of little significance to the study because these engineered designs are usually governed by wind loadings.

Approximately eight archetype performance groups were initially identified, as illustrated in Table 1, then engineered in conformance with the 2012 IBC, ASCE/SEI 7-10 and standard industry practices. Wood and steel deck diaphragms were evaluated. Nail fasteners were judged appropriate for the wood structural panel diaphragms. Recognizing that steel deck diaphragm performance will be different depending upon the deck-to-framing connectors as well as the side-lap seam connectors, several performance sub-groups were studied involving whether welds, pins, screws, punches, or combinations were used. The engineered designs for the various performance groups were numerically modeled for P-695 evaluation.

Table 1. Archetype Performance Groups

Performance Group	1	2	3	4	5	6	7	8	9	10	11
Diaphragm Construction <sup>1</sup>	Wood Structural Panel				Steel Deck						
Seismic Design Load Level <sup>2</sup>	SDC $D_{max}$		SDC $C_{max}$		SDC $D_{max}$				SDC $C_{max}$		
Building Size <sup>3</sup>	Large	Small	Large	Small	Large		Small		Large		Small
Connectors <sup>4</sup>	Nail	Nail	Nail	Nail	Weld/BP	PAF's & Screws/ Screws	Weld/BP	PAF's & Screws/ Screws	PAF's & Screws/ Screws	Weld/Weld	PAF's & Screws/ Screws

1. B-type steel deck and oriented strand board wood structural panels considered. Wood framed and “hybrid” framed roof structures are expected to have similar seismic response and thus share performance groups.
2. Seismic design load levels are in conjunction with ASCE/SEI 7-10 Seismic Design Categories.
3. Building sizes vary from 100-ft to 400-ft diaphragm horizontal clear spans. Large Buildings: 400ft x 200ft (2:1), 200ft x 400ft (1:2), 400ft x 400 ft (1:1); Small Buildings: 200ft x 100ft (2:1), 100ft x 200ft (1:2), 100ft x 100 ft (1:1)
4. Steel deck is fastened with a combination of “frame/side-lap” connectors indicated. Button punch (BP) side-laps are replaced with welded side-laps in diaphragm zones where shear demands are high. PAF's indicate the use of power actuated fasteners.

## Numerical Model Development

Accurately modeling large flexible diaphragms is a very complex and numerically demanding process. With the large numbers of fasteners interacting with the decking, each at a different inelastic state, the tracking of individual fastener performance and its impact on collapse performance is a monumental task. Adding to the burden is the suite of time-histories being evaluated incrementally towards system failure for each archetype performance group.

Conducting a P-695 validation study using a traditional finite element model on a full structure is impractical considering the huge amount of computational time it would require.

To streamline the process, a two dimensional numerical framework for nonlinear dynamic analyses was developed using a three step sub-structuring approach, and is the subject of a companion paper [14, 15]. This approach begins with assembling the hysteretic responses of the diaphragm connectors, then building an inelastic model of the diaphragm discretized into horizontal slices with the connectors, and lastly assembling the various diaphragm slices into an overall simplified building model complete with in-plane and out-of-plane inelastic wall responses, and second order ( $P-\Delta$ ) effects. This model was found to be in very good correlation with experimental and analytical studies available in the literature, capturing the nonlinear response of RWFD buildings. This numerical framework is simplified enough for researchers to efficiently conduct a large number of nonlinear time-history analyses in a timely manner, and has the potential for practitioners to better investigate new and existing RWFD buildings.

### **Findings**

After completing 44,000 non-linear time history dynamic analyses across the archetype performance groups, some interesting results have been observed. For the archetypes studied, the results indicate that inelastic seismic response of the modeled RWFD buildings is clearly within the diaphragm instead of the in-plane walls, but more importantly the inelastic behavior is concentrated more towards the diaphragm boundaries near the parallel shear walls. Even though the diaphragm design capacity was intentionally stepped down to efficiently follow the shear demand reduction towards the diaphragm interior as is often done in practice, the modeling results indicate that the inelastic response of the diaphragm is still concentrated near its shear wall boundary. Because the inelastic behavior is not well distributed across the diaphragm, the localized inelastic response near the boundary is quickly overwhelmed by its limited ability to dissipate large amounts of energy, and leads to global building failure. This phenomenon was observed in both the steel and wood deck diaphragm models, and the FEMA P-695 collapse margin ratio evaluations were often problematic.

A potential design methodology to improve performance is to intentionally weaken the diaphragm's interior areas below current code-based force demands to better distribute the energy absorbing inelastic behavior. The results of the numerical modeling of this weakening approach are promising. This approach of relative weakening assists in protecting the perimeter boundary areas from excessive inelastic demand and increases the margin against collapse. A comparison of the inelastic distribution between an efficient code-based diaphragm design and a weakened one is shown in Fig. 1. The collapse margin ratios used to evaluate this design approach under FEMA P-695 were found to improve significantly with the intentional weakening.



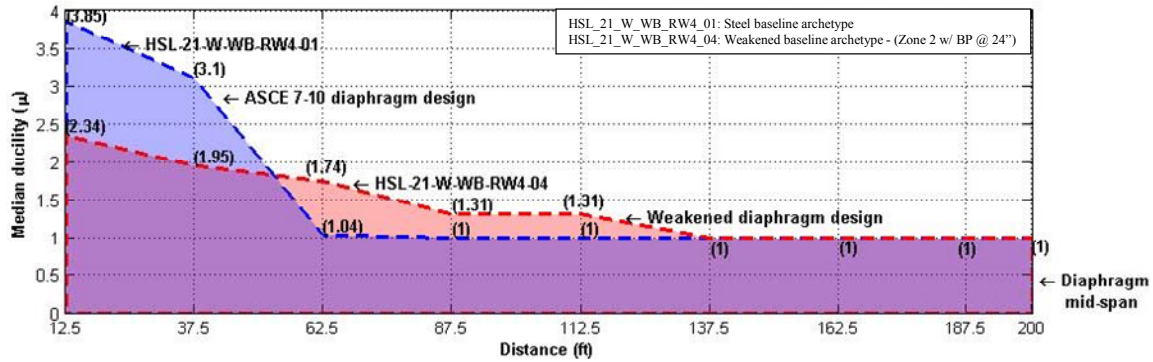


Figure 1. Median ductility distribution at MCE across one-half of diaphragm length of a steel deck archetype building in Performance Group 5 [15].

One problematic issue with this approach is the necessary weakening of the diaphragm would be difficult to enforce in practice. Often in smaller buildings with low shear demands a minimum fastener size at maximum spacing is provided uniformly across the entire diaphragm, thus not accessible to intentional weakening. In these situations, an option to consider is providing overstrength to the boundary areas to mitigate the inelastic behavior. In this approach, the diaphragm boundary areas would be designed for a lower R-value compared with the interior areas, or alternatively the boundary areas are designed with an applied overstrength factor. In other words, instead of relative weakening of the interior areas, a strengthening of the boundary areas is pursued. Another series of 47,520 non-linear time history dynamic analyses were conducted on performance groups using this new design approach, which consisted of designing for the spectral acceleration associated with an estimated diaphragm period, weakening the interior diaphragm portions with a response modification factor  $R_{diaph} = 4.5$  (instead of 4), and strengthening the diaphragm boundaries (a width of 10% of the diaphragm span) with 50% more capacity. With this new proposed design approach, the P695 evaluation showed a significant improvement in the adjusted collapse margin ratio. Table 2 illustrates the significant increase in the adjusted collapse margin ratio (ACMR) of five steel deck performance groups under existing code-based design practices compared with this new proposed design approach.

Table 2. Comparison of P695 Results of the Current Code-based Design Approach with a New Proposed Design Approach for Select Performance Groups

Performance group ID	Design Configuration						Collapse margin parameters		Pass/Fail
	Design (E:existing N:new)	Building Size	Diaphragm Construction	Seismic SDC	Framing connectors	Sidelap connectors	ACMR	Acceptable ACMR	
PG-5	E	Large	Steel (WR Deck)	Dmax	Welds	Welds & button punches	1.71	2.30	Fail
	N	Large	Steel (WR Deck)	Dmax			2.73		Pass
PG-6	E	Large	Steel (WR Deck)	Dmax	PAF/screws	Screws	1.87	2.30	Fail
	N	Large	Steel (WR Deck)	Dmax			2.75		Pass
PG-7	E	Small	Steel (WR Deck)	Dmax	Welds	Button punches	2.23	2.30	Fail
	N	Small	Steel (WR Deck)	Dmax			2.82		Pass
PG-8	E	Small	Steel (WR Deck)	Dmax	PAF/screws	Screws	1.92	2.30	Fail
	N	Small	Steel (WR Deck)	Dmax			2.40		Pass

Using the special numerical model framework developed for this project, a separate side study was also conducted to investigate the magnitude of the wall anchorage forces for various median spectral accelerations at the fundamental period of the building. Data was collected from the model indicating the attachment force between the out-of-plane wall panels and the diaphragm. The walls as modeled were simply supported at their tops and bottom and were permitted to crack and yield out-of-plane, but another study was also conducted to observe the effects if the simply supported walls remained rigid. Results from a large rectangular building incorporating a steel roof diaphragm are shown in Fig. 2 and clearly demonstrate the softening of the diaphragm caused by the inelastic response at high spectral accelerations. Higher wall anchorage forces at the ends of the rectangular building were observed when excited in the long direction. This phenomenon is likely caused by the diaphragm's inherent longitudinal overstrength due to the transverse loaded shear design.

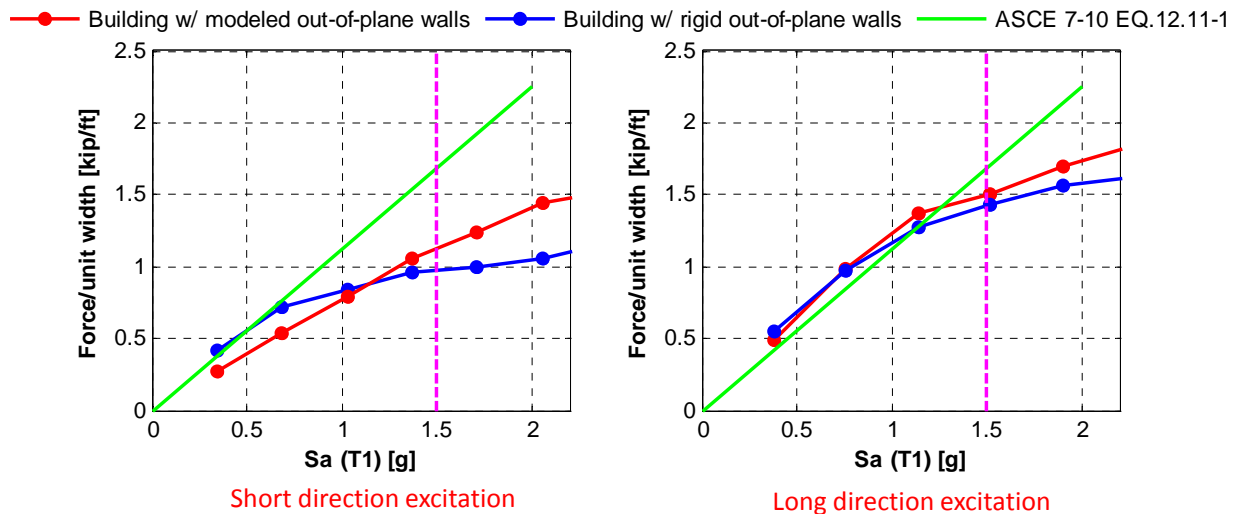


Figure 2. Wall anchorage forces at diaphragm mid-span associated with Performance Group 1 [Table 1]. Walls are 9¼-inch thick concrete, 30-feet high to roof with 3-foot parapet.

The current ASCE/SEI 7-10 wall anchorage design force is also shown in Fig. 2. The intent of this force is to design for maximum expected force levels without reliance on anchorage system ductility [4]. The ASCE/SEI 7-10 anchorage force levels are appropriate in the long direction and conservative in the short direction of excitation based on this archetype example.

## Conclusions

Despite how common RWFD structures are, their behavior is not well represented within the current building code provisions. Past earthquake damages have led to improved code provisions for wall anchorage; but it is feared that the inelastic response will now transfer into the main diaphragm where there may be limited ability to accommodate the necessary inelastic demand. A two dimensional numerical model framework and methodology validation was developed to efficiently evaluate the unique seismic performance of RWFD buildings and evaluate potential design methodologies to mitigate poor collapse margins driven by performance of the flexible

diaphragms.

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